# Appendix A

## Grounding Requirements

The tower base and guy anchors shall be grounded in accordance with standards of good engineering practice.

At each guy point at least one ¾" ground rod will be driven to a depth of thirty (30) feet or refusal and connected to each guy wire by 250 MCM bare copper wire utilizing upson walton wire rope kits (or better) as shown in the attached PDF drawing. The connection of the 250 MCM bare copper wire to the ground rod shall be via cadweld (exothermic connection).

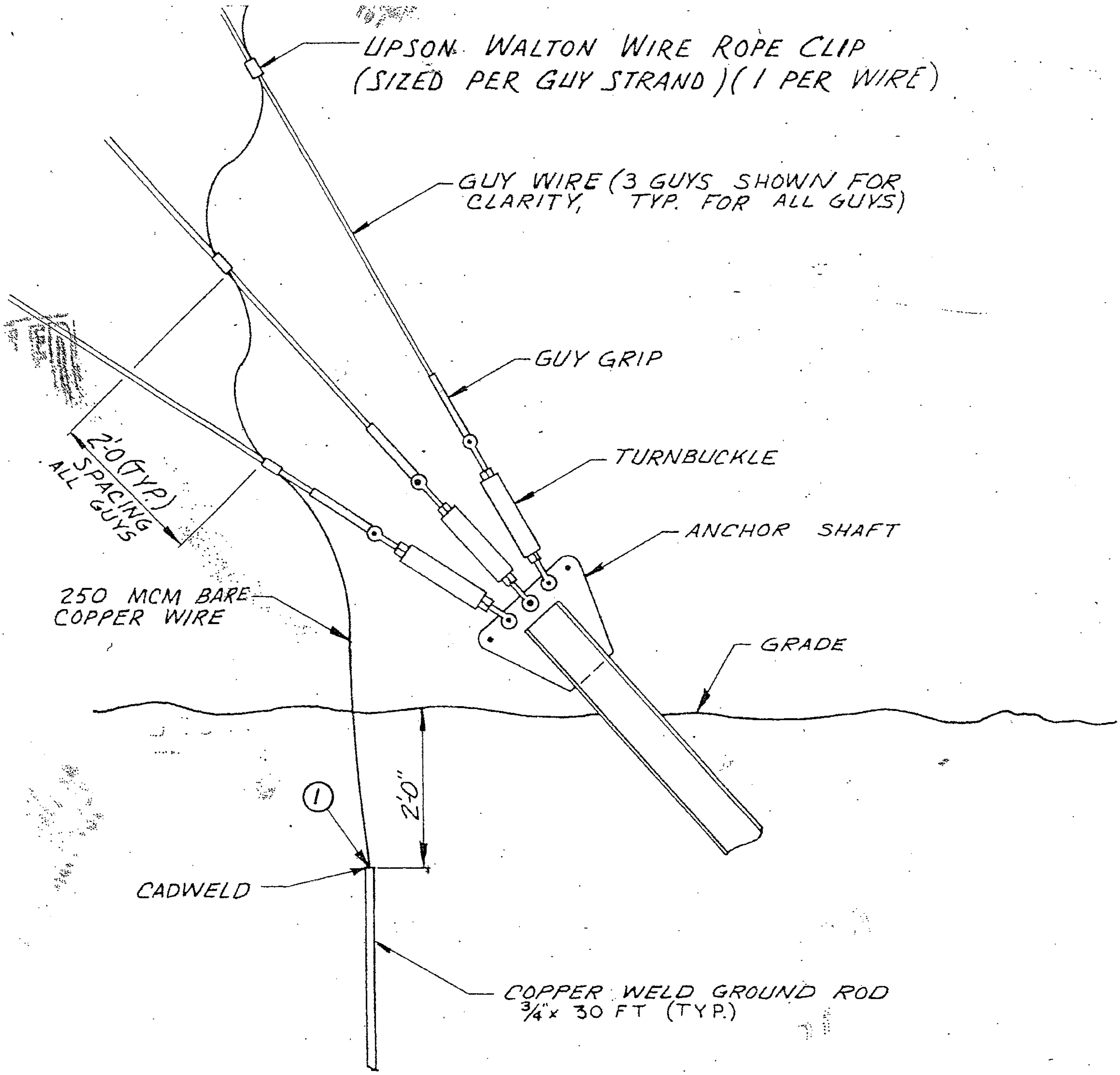
The tower shall be of the pivot base design and shall be grounded by at least five (5) ¾" ground rods arranged in a ring configuration with at least a 12 foot radius to the base of the tower and driven to at depth of thirty (30) feet or refusal. The ground rods will then be cadwelded in a ring configuration utilizing 250 MCM bare copper wire. The ground ring will then be connected to the base plate of the tower by cadwelded 250 MCM bare copper wire from each of the five (5) ground rods.

The tower base grounds shall be cadwelded to the station and utility grounds by 250 MCM bare copper wire.

The tops of all ground rods, the ground ring, and all connecting wires are to be buried at least 24 inches below grade.

All splices, junctions and connections are to be cadwelded. All ground rods to be copper or copperweld, ¾" dia.

Wires are to be routed as directly as possible avoiding sharp bends or unnecessary splices.


UPSON WALTON WIRE ROPE CLIP (SIZED PER GUY STRAND)(1 PER WIRE)
GUY WIRE (3 GUYS SHOWN FOR CLARITY, TYP. FOR ALL GUYS)
GUY GRIP
TURNBUCKLE
ANCHOR SHAFT
2'-0" (TYP.) SPACING ALL GUYS
250 MCM BARE COPPER WIRE
GRADE
2'-0"
1
CADWELD
COPPER WELD GROUND ROD ¾"x 30 FT (TYP.)

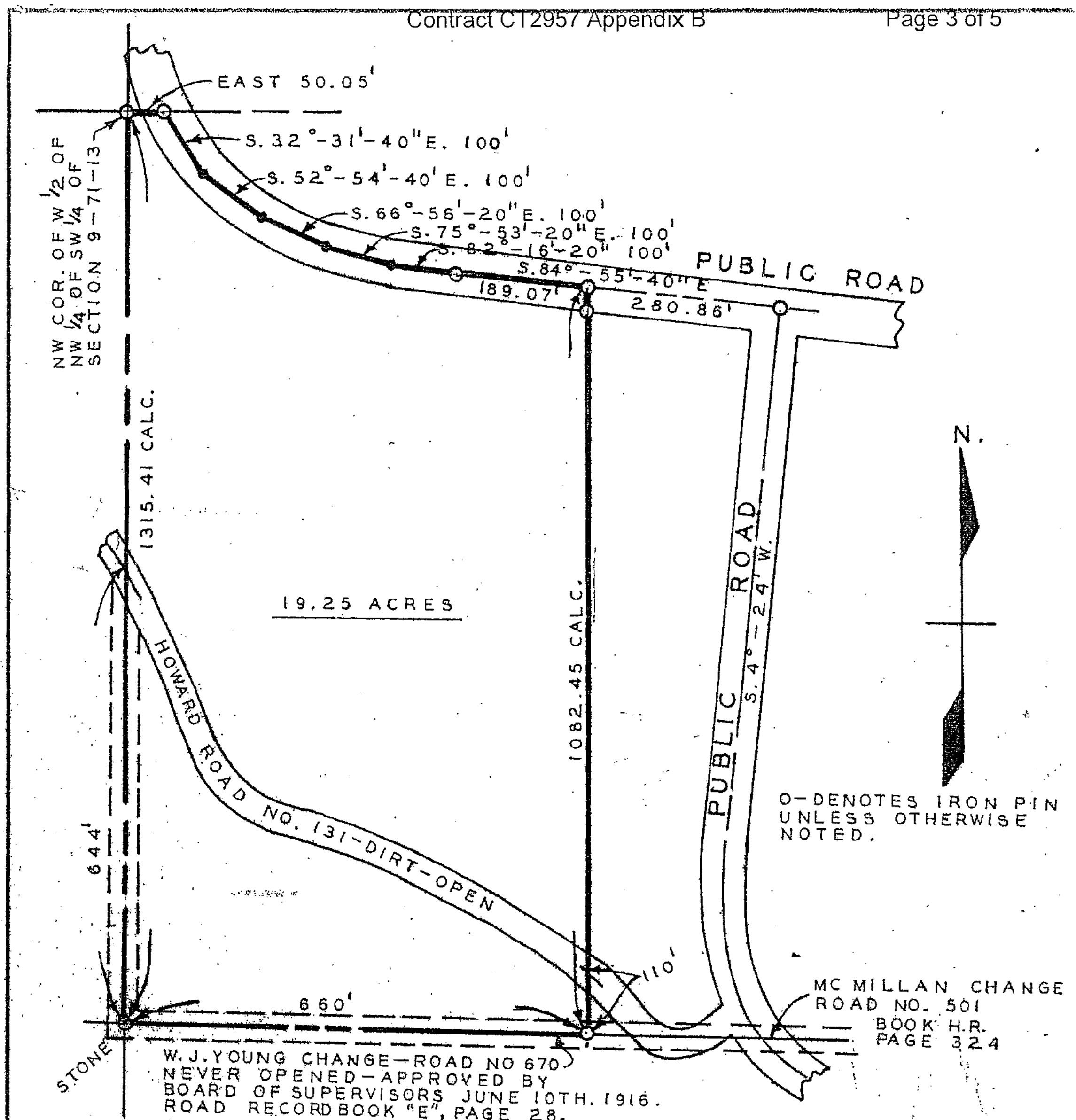
# Appendix B
# Site Drawings

# Please Note:

Road names have changed from those listed on attached surveyor documents.

McMillan Change Road No. 501 is now known as Miller Chapel Road.
Howard Road NO. 131 Dirt is now known as 53$^{rd}$ St.

53$^{rd}$. St. is still dirt and functions as the drive into the tower site only.




## C E R T I F I C A T E

I hereby certify that the above is a plat of a survey as made by me or under my direct personal supervision, that I am a duly Registered Land Surveyor under the laws of the State of Iowa and that the following is a true and correct description of the tract of land surveyed.

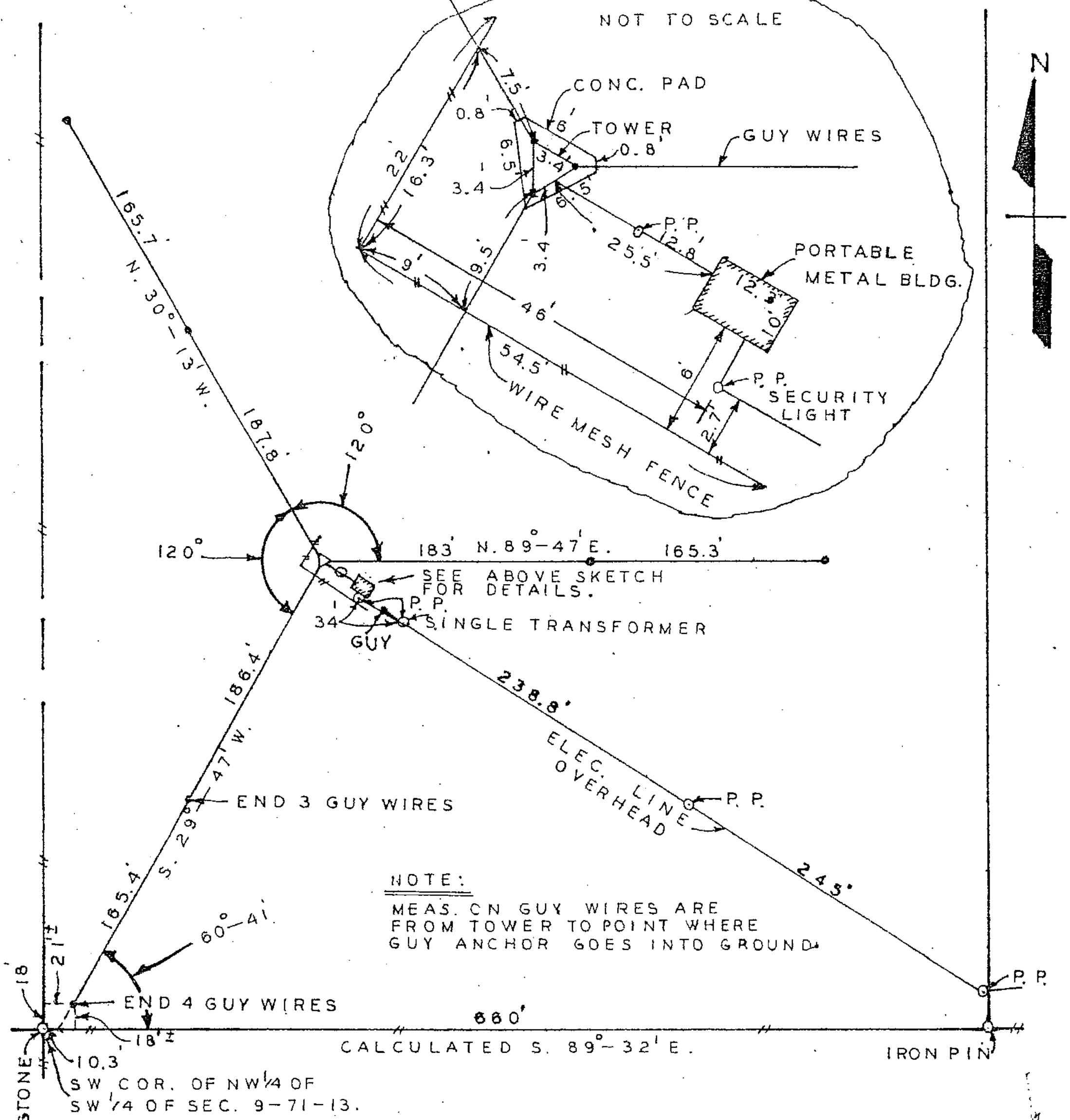
"All that part of the West One-half (W$\frac{1}{2}$) of the Northwest Quarter (NW$\frac{1}{4}$) of the Southwest Quarter (SW$\frac{1}{4}$) of Section Nine (9), Township Seventy-one (71) North, Range Thirteen (13) West of the Fifth (5th) Principal Meridian, in Wapello County, Iowa that lies South of the centerline of the existing Public Road as now located along the Northerly side of said tract of land, containing 19.25 Acres. The centerline of the said Public Road being described as follows, to-wit: Beginning at a point on the centerline of the Public Road that is 50.05 ft. East of the Northwest corner of the W$\frac{1}{2}$ of the NW$\frac{1}{4}$ of the SW$\frac{1}{4}$ of said Section 9; thence running Southeasterly along the following chords of the irregular curve of the Public Road, S. 32°- 31'- 40" E., a dist. of 100 ft.; thence S. 52°- 54'- 40" E., a dist. of 100 ft.; thence S. 66°- 56'- 20" E., a dist. of 100 ft.; thence S. 75°- 53'- 20" E., a dist. of 100 ft.; thence S. 82°- 16'- 20" E., a dist. of 100 ft. to the point of tangency of the said irregular curve; thence S. 84°- 55'- 40" E. along the centerline of the Public Road, a dist. of 189.07 ft. to a point on the East line of the W$\frac{1}{2}$ of the NW$\frac{1}{4}$ of the SW$\frac{1}{4}$ of Section 9."

*Lewis E. Graham Jr.*
Lewis E. Graham, Jr. L.S.
~~Reg. No. 3955~~

**LEWIS E. GRAHAM, JR.**
Professional Land Surveyor
OTTUMWA IOWA

LEE ENTERPRISES, INC.

DATE 13 FEB 1969 | SCALE 1" = 200' | NO.




## CERTIFICATE

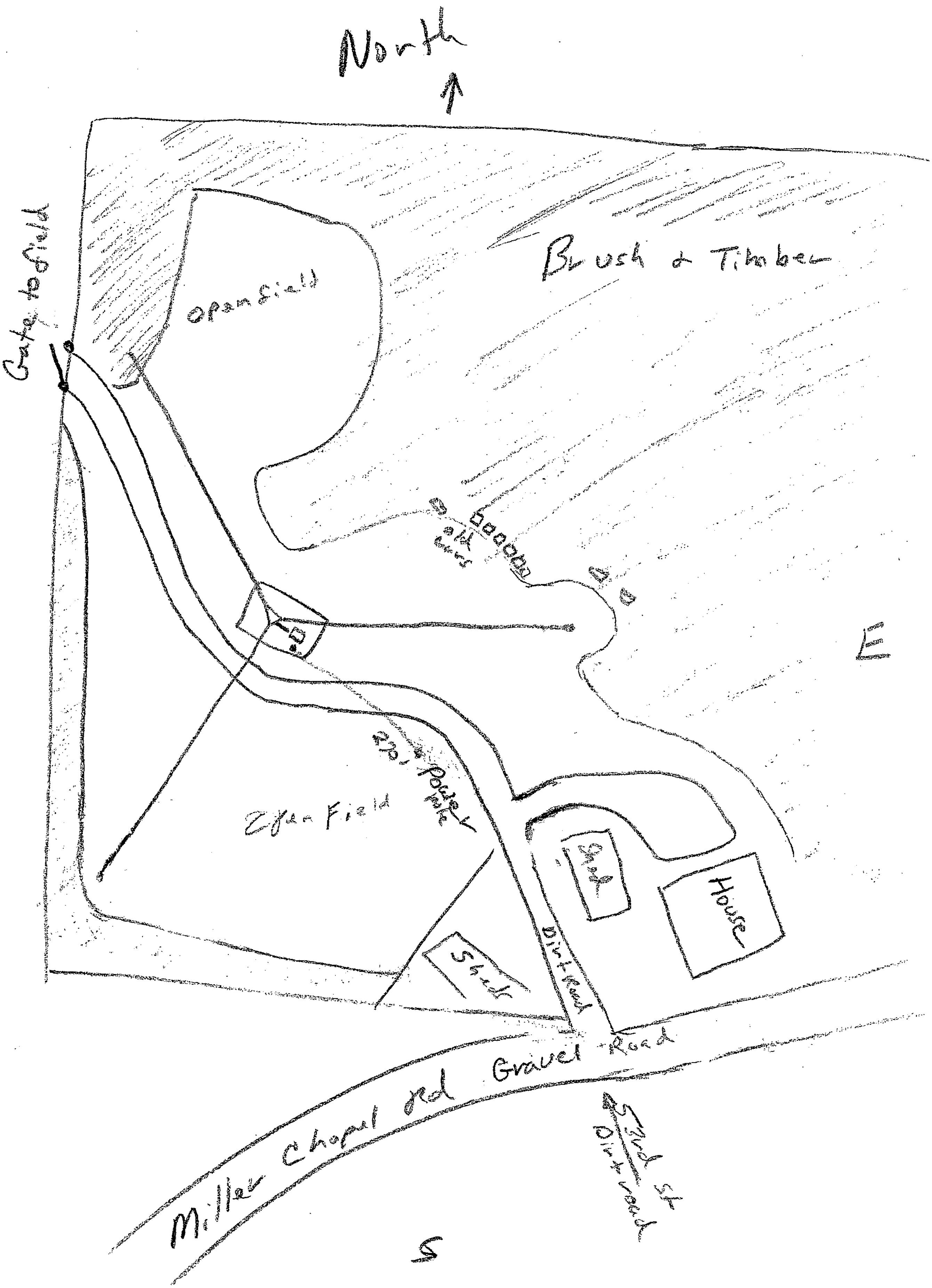
I hereby certify that the above is a plat of a situation survey as made by me or under my direct personal supervision, that I am a duly Registered Land Surveyor under the laws of the State of Iowa, that the plat correctly shows the improvements as now located on the property and that the description of the above said property can be found on my survey plat No. 69-8, dated 13 Feb. 1969 for Lee Enterprises, Inc., all to be true and correct to the best of my knowledge and belief.

Lewis E. Graham, Jr.
Lewis E. Graham, Jr. R.L.S.
Reg. No. 3955

| | |
|---|---|
| LEE ENTERPRISES, INC.<br>TRANSLATOR TOWER SITE<br>I. P. B. N. | LEWIS E. GRAHAM, JR.<br>Professional Land Surveyor<br>OTTUMWA IOWA |
| DATE: 11 FEB. 1977 | SCALE: 1" = 100' NO. 77-7 |

Ottumwa Translator Tower site
Not to scale





# Appendix C
# Geotechnical Report

# TEAM *Services*

Soil, Environmental and Material Consultants

# SUBSURFACE EXPLORATION
# IOWA PUBLIC TELEVISION TOWER
# 3½ MILES SE OF OTTUMWA, IOWA
# TEAM NO. 1-1571
# MAY 24, 2005

**TEAM *Services***
Soil, Environmental and Material Consultants

May 24, 2005

Iowa Public Television Tower
PO Box 6450
Corporate Drive
Johnston, IA 50131

Attn: Gary McMillen

Re: Subsurface Exploration
Iowa Public Television tower
3½ miles southeast of Ottumwa, Iowa
TEAM No. 1-1571

Dear Mr. McMillen:

We have completed the subsurface exploration for the proposed Iowa Public Television tower near Ottumwa, Iowa. The accompanying geotechnical report presents the findings of the subsurface exploration and our recommendations concerning foundation design and construction for the proposed facility.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service to you in any way, please do not hesitate to contact us.

Sincerely yours,

TEAM Services

Shiping Yang, Ph.D., P.E.
Senior Project Engineer

Robert E. Doss, P.E.
Principal

Cc: Andy Hills, Central Tower

I hereby certify that this engineering document was prepared by me or under my direct personal supervision and that I am a duly licensed professional engineer under the laws of the State of Iowa.

Robert E. Doss, P.E. Iowa License No. 12543 Date: 05/24/05
My license renewal date is December 31, 2006.
Pages covered by this seal: ALL

LICENSED PROFESSIONAL ENGINEER
ROBERT E.
DOSS
12543
IOWA

333-H SW 9th Street • Des Moines, IA 50309-4440 • ph.: 515-282-8818 • fx: 515-282-8741 • email: staff@teamsvcs.com
Des Moines • Ft. Dodge • Cedar Rapids • Mason City • Davenport

*Subsurface Exploration* TEAM *Services*
*Iowa Public Television tower*
*3½ miles southeast of Ottumwa, Iowa*
*TEAM No. 1-1571*
*May 24, 2005*

**TABLE OF CONTENTS**

*Subsurface Exploration*
*Iowa Public Television tower*
*3½ miles southeast of Ottumwa, Iowa*
*TEAM No. 1-1571*
*May 24, 2005*

## PROJECT INFORMATION

Project information has been provided by via email on February 24, 2005 from Gary McMillen of IPTV to our Mr. Colby Cunningham, P.E. Included in the email were a sketch of the site, a scanned site survey by Lewis Graham Jr., P.L.S., dated February 13, 1969, a drawing by the same author and date showing the original tower location, and a drawing with notes entitled "Soil Information Data, Iowa Public Television – 478' Option by Central Tower" dated February 22, 2005. The project will include construction of a 478 foot tall guyed tower about 3½ miles southeast of Ottumwa, Iowa. The new tower will replace the existing tower at the site.

## SITE CONDITIONS

The project site is located north of 53rd Street on the west side of Miller Chapel Road approximately 3½ miles southeast of Ottumwa, Iowa. The site is currently a gently sloping grass and wooded field. Surface soils were able to support our ATV-mounted auger drill rig without difficulty.

## FIELD EXPLORATION

Boring holes were laid out on the site by others. A total of 7 borings were laid out with one boring at the tower base location and one at each of the anchor locations. Because the tower center boring was located within the fenced area and inaccessible to our drill rig, this boring was moved outside the fence. The approximate boring locations are indicated on the Boring Plan in the Appendix. The existing ground surface elevations at the boring locations were not obtained. The locations of the borings should be considered accurate only to the degree implied by the means and methods used to define them.

Our drilling equipment consisted of an ATV-mounted auger drill rig. The borings were made by mechanically twisting a continuous flight hollow stem steel auger into the soil. At assigned intervals, the center drive bit of the auger was removed and soil samples were obtained.

Representative samples were obtained using thin-walled (Shelby) tube and split-barrel sampling procedures in general accordance with ASTM Specifications D-1587 and D-1586, respectively. In the thin-walled tube sampling procedure, a thin-walled, seamless steel tube with a sharp cutting edge is pushed hydraulically into the ground to obtain relatively undisturbed samples of cohesive or moderately cohesive soils. In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon is driven into the ground with a 140-pound hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the standard penetration resistance value. These values are indicated on the boring logs at the depths of occurrence. Bulk samples were also obtained the depth of significant strata change. The locations of the bulk samples are indicated on the appropriate boring logs or on the appropriate test sheets. These locations should be considered approximate, because the correlation between the depth of the augers at the time of sampling and the sample content itself is approximate. The smearing action of the soil traveling up the auger flights tends to mix the soil and also scrapes some soil off the side of the boreholes. The samples were tagged for identification, sealed and returned to the laboratory for testing and classification.

An automatic hammer was used in the Standard Penetration Tests performed for the borings at this site. In the automatic hammer system, the cathead and rope used traditionally in the manual test procedure is replaced with an automatic lifting mechanism for the 140 pound driving weight. The reduction in system friction with the automatic hammer system results in a significant increase in the driving energies. This results in significantly greater driving efficiencies and a corresponding decrease in the number of blows in the Standard Penetration Test results. We have taken the driving efficiency of the automatic hammer system into account when analyzing this data.

Field logs of the borings were prepared by the drill crew. These logs included visual classifications of the materials encountered during drilling, as well as the driller's interpretation of the subsurface conditions between samples. Final boring logs included with this report represent an interpretation of the field logs and include modifications based on laboratory observation and tests of the samples.

## LABORATORY TESTING

Based on the driller's field records and examination of the samples in the laboratory, a soils testing program was developed to collect more information about the soil conditions at the site. The following is a brief description of the specific tasks completed for this project.

**Natural Moisture Content** -- The natural moisture content of selected samples was determined in general accordance with ASTM D2216. The moisture content of the soil is the ratio, expressed as a percentage, of the weight of water in a given mass of soil to the weight of the soil particles. The results are presented on the boring logs at the depths from which the samples were obtained.

**Unit Weight** -- In the laboratory, selected undisturbed samples of the site soils were measured and weighed to determine gross weight and volume of the samples. Where possible, the samples are placed in a template and trimmed at each end to fit the template. The moisture content of each specimen was then determined, and the dry unit weight was calculated. The results of these tests are also presented on the boring logs at the appropriate sample depths.

**Unconfined Compressive Strength** -- Selected cohesive soil samples obtained with 3-inch diameter Shelby tubes were tested in the laboratory to determine their unconfined compressive strength in general accordance with ASTM D2166. In this procedure, sections of the Shelby tube samples were trimmed to fit into a 2.875 inch diameter by 5.75 inch high template and placed, without any confinement, in a triaxial load frame and tested for compressive strength with a controlled rate of strain. The peak stress on the samples, in psf, is reported on the boring logs at the depth from which the samples were obtained. A calibrated hand penetrometer was used to estimate the approximate unconfined compressive strength of the remaining samples. The calibrated hand penetrometer has been correlated with unconfined compression tests and provides a better estimate of soil consistency than visual examination alone.

As part of the testing program, the samples were classified in the laboratory based on visual observation, texture and plasticity. The descriptions of the soils indicated on the boring logs are in accordance with the enclosed *General Notes* and the *Unified Soil Classification System*. Estimated group symbols according to the *Unified Soil Classification System* are given on the boring logs. A brief description of this classification system is attached to this report.

## SUBSURFACE CONDITIONS

Subsurface conditions encountered during this exploration are indicated on the individual boring logs. Based on the results of the borings, subsurface conditions on the project site can be generalized as follows.

From the ground surface at the site, wind-deposited soils, or loess were encountered. The loess soils at the site consisted of silt and lean clay. Loess soils include primarily silt sized particles but a lesser fraction of clay dominates the behavior of the materials. Loess soils have typically not experienced significant overburden pressures beyond the weight of the soil above them; below the zone of soil affected by seasonal wet/dry cycles (where some preconsolidation by desiccation has occurred), the loess is often near-normally consolidated. The loess soils at the site were typically medium stiff to very stiff in consistency. These materials extended to about 7 to 14 feet below existing grades.

Beneath the wind-deposited soils at the site, soils developed from ancient (pre-Illinoian) glacial activity were encountered. These materials, which are usually called glacial tills, are typically stiff to very stiff. Soils derived from glacial till were deposited during the advance or retreat of continental glacial ice sheets which covered this area many thousands of years ago. The glacial till soils are more or less unsorted soil deposits consisting of a homogeneous mixture of sand, silt and clay, with the engineering properties of the soil being controlled by the clay fraction. Glacial till has usually been heavily preconsolidated by the glacial ice sheet from which it was derived. This preconsolidation compacts the soil and gives it superior properties for foundation support. The glacial till soils at the site were typically lean clay or fat clay with stiff or very stiff consistency and extended to the maximum depth explored of about 40½ feet below existing grade.

The glacial till deposits were once at the ground surface in this area in the time period before the loess deposition occurred. During this time, the glacial till appears to have developed a topsoil horizon and a weathered zone of fat clay at its surface. The buried topsoil horizon is still present in some locations and is termed a "paleosol".

Cobbles and boulders were not noted in our boring. However, glacial soils were encountered at the site, and these materials often contain cobbles and boulders. The possibility of their presence should be considered where excavations or grading operations at the site advance into the glacial soils.

The above descriptions provide a general summary of the subsurface conditions encountered. The attached Boring Logs contain detailed information recorded at each boring location. These Boring Logs represent our interpretation of the field logs based on engineering examination of the field samples. The lines designating the interfaces between various strata represent approximate boundaries and the transition between strata may be gradual. It should be noted that the soil conditions will vary between the boring locations.

## GROUNDWATER CONDITIONS

The borings were monitored while drilling and after completion for the presence and level of groundwater. At the time of our exploration, groundwater was not observed within the borings during drilling or after the drilling was completed. Groundwater levels were rechecked at the site after about 24 hours the drilling was completed. Groundwater was observed within Borings 1, 3 and 7 at about 17 to 22 feet below existing grades. These water level observations provide an approximate indication of the groundwater conditions existing on the site at the time the borings were drilled. Due to the low permeability of the cohesive soils encountered in the borings, a relatively long period of time may be necessary for a groundwater level to develop and stabilize in a borehole. Longer term monitoring in cased holes or piezometers would be required for a more accurate evaluation of the groundwater conditions.

Groundwater levels may fluctuate several feet with industrial, seasonal and rainfall variations and with changes in the water level in adjacent drainage features. Normally, the highest groundwater levels occur in late winter and spring and the lowest levels occur in late summer and fall.

## CONCLUSIONS AND RECOMMENDATIONS

### General

The silty loess soil on the site is highly susceptible to disturbance from construction activity, particularly if the soil has a high natural moisture content or is wetted by surface water or seepage. Care should be taken during excavation and construction of footings to minimize disturbance of the bearing soils. Heavy equipment traffic directly on bearing surfaces should be avoided in wet silty soils.

### Site Preparation

Little grading is anticipated at this site. If grade changes are to occur, TEAM Services should be contacted and provided with detailed site grading information so that we can review the grading information and provide additional recommendations, if needed.

Fill and backfill placed for support of structures should consist of approved materials which are free of organic matter and debris. Bricks, rocks, or other solid pieces with a maximum dimension of 3 inches or larger should not be placed in the new fill, nor should organic material be used. Cohesive or granular soil may be used for fill. Most of the soils encountered at the site appear to be suitable for use as new fill.

Structural fill placed in building and parking areas should be placed and compacted in lifts of 9 inches or less in loose thickness and compacted to at least 95 percent of the material's standard Proctor maximum dry density (ASTM D-698) for cohesive soils and 98 percent for cohesionless soils. Fill should be placed at a moisture content between 2 percent below and 4 percent above the material's optimum moisture content (also based on ASTM D-698) for cohesive soils and within 3 percent of the optimum moisture content for cohesionless soils. Sufficient density tests should be performed on each lift of fills to help verify the adequacy of the compaction levels obtained. Upon completion of the filling operation, care should be taken to maintain the subgrade moisture content prior to placement of the foundation. If the subgrade should become desiccated, frozen or otherwise disturbed, the affected material should be removed or these materials should be scarified, moistened, recompacted and retested prior to concrete placement. As a general guideline, fills which dry to a moisture content less than 2/3 of their optimum moisture content as determined by the Standard Proctor Test (ASTM D-698) in their upper 2 inches are candidates for reconditioning as described above.

**Shallow Foundation Design**

The proposed tower may be supported on a system of shallow spread footing foundations bearing in the natural stiff lean clay or lean to fat clay encountered in our borings or on newly placed compacted and tested suitable granular fill, which extends to the suitable natural soil.

Shallow footing foundations may be designed for a maximum net allowable bearing pressure of 2500 psf. This is the net pressures in excess of the minimum adjacent overburden pressure and applies to the maximum dead load plus the sustained live load. The bearing pressures may be increased 30 percent for the effects of transient loads such as earthquake or wind loads. Footings should extend at least 42 inches below lowest adjacent finished grade for frost protection.

**Shallow Foundation Construction**

Foundation construction at this site should be rapid, with excavations completed and filled with concrete on the same day and in as short a period as possible during the day. This measure is necessary to minimize disturbance to the foundation bearing surfaces and loss of moisture where fat clay is exposed. We recommend that the base of all footing excavations be observed and tested by the geotechnical engineer prior to placement of concrete. Where disturbed soil or loose or soft zones are encountered, the foundation excavation should be deepened to reach suitable natural soils. The footing could then bear at the lower level on approved natural soil or at the original design level on properly compacted structural backfill. Refer to Figure 1 for an illustration of the overexcavation and backfill procedure.

The depth to suitable natural soils (indicated as "D" in Figure 1) should be determined in the field by TEAM Services. Overexcavation for compacted backfill placement below footings should extend beyond all edges of the footings at least 8 inches laterally per foot of overexcavation depth below design footing level. The overexcavation should then be backfilled up to the footing base elevation with suitable fill prepared in accordance with the recommendations of this report.

**Lateral and Uplift Force Resistance**

Foundations for the guy anchors are subjected to some lateral and uplift forces. The foundations should be sized to resist the anticipated forces without excessive deflection and displacement.

Lateral forces on the concrete will be resisted by the friction between the base of the foundation and the underlying soils and passive earth pressures. A coefficient of 0.3 could be reasonably assumed for evaluating ultimate frictional resistance to sliding at the foundation-soil contact. This coefficient should be used with minimum dead load as the normal force. The buoyant weight should be considered in calculation of the minimum weight of all below-grade structural elements. A passive earth pressure coefficient of 3 could be reasonably assumed for evaluating ultimate lateral resistance of the soil against the side of the foundation where this is a permissible condition. This passive earth pressure should be divided by a safety factor of at least 2 to limit the amount of lateral deformation required to mobilize the passive resistance. In order to calculate passive soil resistance, the buoyant unit weight of the soil should be utilized. A reasonable value for the buoyant unit weight of the soils at the site is 55 pcf, considering that the groundwater level at the site may rise to near existing grade at some times during the year. For transient load calculations, the total unit weight of the soils of 120 pcf can be reasonably assumed. The contribution to passive resistance of the frost affected materials in the upper 4 feet at the site should be limited to solely the weight of this soil. This can be accomplished by modeling this depth of soil as a uniform surcharge load equivalent to the weight of the soil in the analysis.

Uplift resistance will be provided by the minimum dead weight of the structure and the foundation elements, plus the weight of the soil above the foundations. The weight of the soil above the foundations and extending outward at a 2 vertical to 1 horizontal slope may be considered as contributing to the uplift resistance of the foundations. This is based on the assumption that the backfill of the foundations will be compacted in accordance with the recommendations of this report for structural fill. The buoyant unit weight of concrete should be considered for the weight of buried concrete anchors. The buoyant unit weight of the soils at the site of 55 pcf is recommended for uplift calculations, considering the maximum water table elevation at the site to be the ground surface elevation. For transient load calculation, the total unit weight of the soils of 120 pcf can be used provided that soils similar to those encountered in our borings are used as fill and compacted in accordance with the recommendations in this report. The maximum upward bearing pressure on horizontal elements of a pedestal type foundation should be checked against a maximum allowable pressure of 2,000 psf on these surfaces.

**Other Soil Parameters**

Other soil parameters that may be needed in foundation design are indicated in the Table 1. The depth of each stratum is an approximate average of the site. The skin friction, end bearing, and passive soil pressure values provided include a safety factor of about 3.

TABLE 1 – Soil Engineering Parameters

| Depth in Feet | | Soil Type | Allowable Skin Friction (psf) | Allowable End Bearing (psf) | Passive Soil Pressure (psf/ft) | φ Friction Angle (o) | C cohesion, (psf) |
|---|---|---|---|---|---|---|---|
| From | To | | | | | | |
| 0 | ≈3½ | Frost Zone | Neglect | Neglect | Neglect | Neglect | Neglect |
| 3½ | 13 | Medium stiff to very stiff lean clay or silt | 250 | 2,500 | 80 | 0 | 1,000 |
| 13 | 40 | Stiff or very stiff fat clay or lean clay | 300 | 6,000 | 70 | 0 | 2,000 |

**Construction Dewatering**

During construction activities, care should be taken to maintain positive drainage at the site. Based on the boring information obtained, it appears unlikely that excavations will be extended below the groundwater level. However, if groundwater is encountered in excavations, then trench drains will likely be required outside the excavation in order to lower the groundwater to at least 2 feet below the proposed excavation level in order to help maintain foundation subgrade stability in the highly susceptible silts and lean clays.

**Site Drainage**

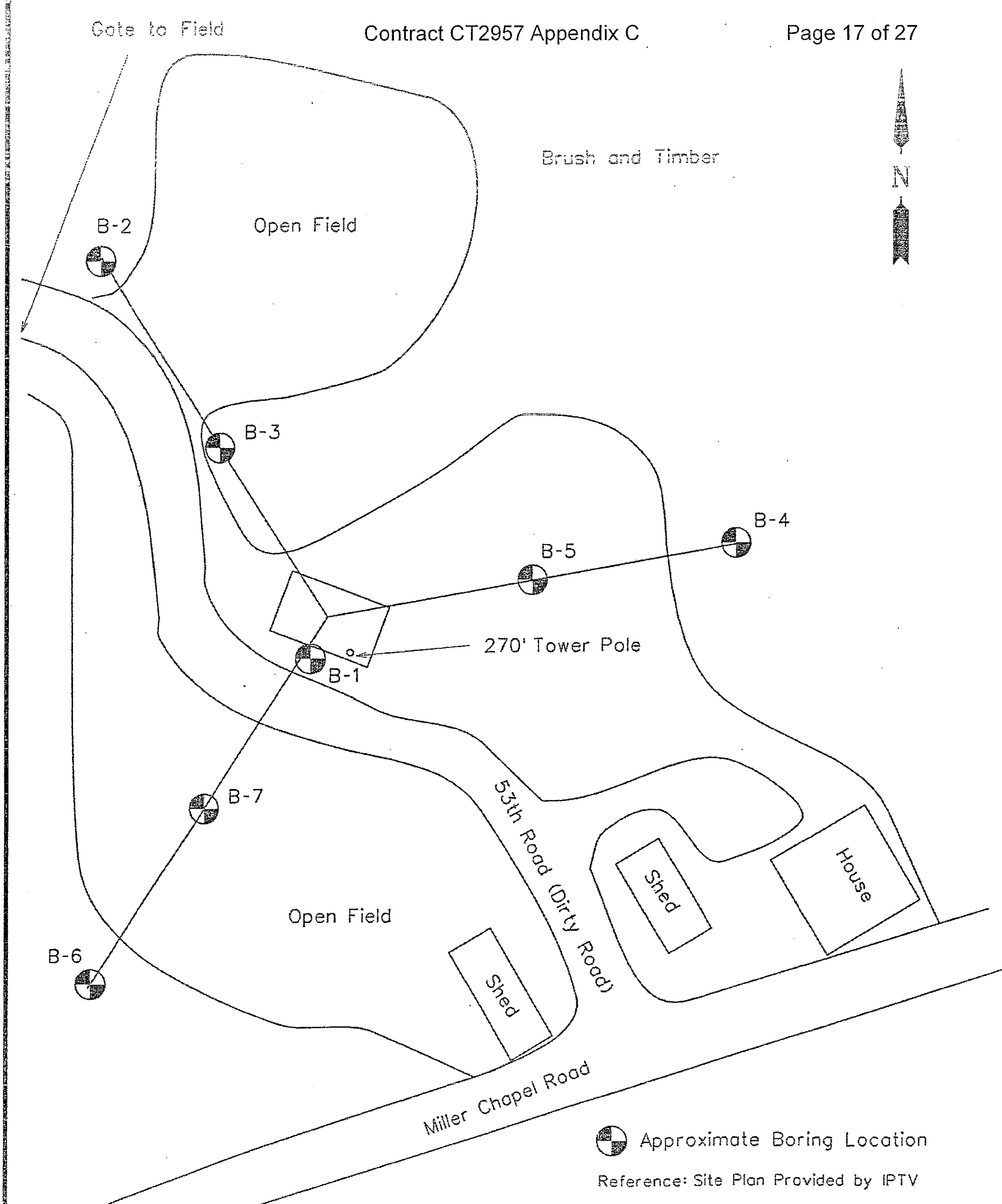
Positive site drainage should be maintained along the perimeter of the tower. Final grades should be established to direct runoff away from foundations. Site grading should direct surface water away from excavations or completed foundations during construction and after site development is completed.

## QUALIFICATION OF REPORT

Our evaluation of foundation support conditions has been based on our understanding of the site and project information and the data obtained in our exploration. The general subsurface conditions utilized in our foundation evaluation have been based on interpolation of subsurface data between the borings. In evaluating the boring data, we have examined previous correlations between soil properties and foundation bearing pressures observed in soil conditions similar to those at your site. The discovery of any site or subsurface conditions during construction which deviate from the data outlined in this exploration should be reported to us for our evaluation. The assessment of site environmental conditions or the presence of pollutants in the soil, rock, and groundwater of the site was beyond the scope of this exploration.

It is recommended that the geotechnical engineer be retained to review the plans and specifications so that comments can be provided regarding the interpretation and implementation of the geotechnical recommendations in the design and specifications. It is further recommended that the geotechnical engineer be retained for testing and observation during the foundation construction phase to help determine that the design requirements are fulfilled.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No other warranty is provided. In the event that any changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report modified or verified in writing by the geotechnical engineer.




| | | |
|---|---|---|
| TEAM Services, Inc<br>333-H S.W. 9th Street<br>Des Moines, Iowa | IOWA PUBLIC TELEVISION TOWER<br>Ottumwa, Iowa<br>Boring Plan | Project No 1-1571<br>05-18-2005<br>No Scale |

# LOG OF BORING NO. 1

Page 1 of 2

| OWNER | ARCHITECT/ENGINEER |
|---|---|
| Iowa Public Television | |
| SITE: Ottumwa, Iowa | PROJECT: Iowa Public Television Tower |

| GRAPHIC LOG | DESCRIPTION | DEPTH (ft.) | USCS SYMBOL | SAMPLES: NUMBER | SAMPLES: TYPE | SAMPLES: RECOVERY | SAMPLES: SPT - N BLOWS / FT. | TESTS: MOISTURE, % | TESTS: DRY DENSITY PCF | TESTS: UNCONFINED STRENGTH PSF |
|---|---|---|---|---|---|---|---|---|---|---|
| | Lean CLAY, trace ferrous staining, olive brown, medium stiff to stiff | | | | HS | | | | | |
| | | 5 | CL | 1 | ST | 19" | | 26.1 | 95 | 1170 3000* |
| | | | | | HS | | | | | |
| | 7.0 Lean CLAY, with sand, dark yellowish brown, stiff | | | | | | | | | |
| | | 10 | CL | 2 | ST | 19" | | 18.5 | 109 | 3500* |
| | 11.0 Lean to fat CLAY, trace sand and calcareous nodules, yellowish brown, stiff | | | | HS | | | | | |
| | | 15 | CL-CH | 3 | SS | 17" | 7 | 21.8 | | |
| | -- becomes very stiff @ about 18' | | | | HS | | | | | |
| | | 20 | CL-CH | 4 | ST | 23" | | 19.0 | 109 | 9000* |
| | ▼ | | | | HS | | | | | |
| | | 25 | CL-CH | 5 | SS | 18" | 11 | 19.4 | | |
| | | | | | HS | | | | | |

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES: IN-SITU, THE TRANSITION MAY BE GRADUAL.

Calibrated Hand Penetrometer*

| WATER LEVEL OBSERVATIONS | | | |
|---|---|---|---|
| WL | ▽ None | WD ▼ 22' | 24 hrs AD |
| WL | None | AD | |
| WL | | | |

**TEAM Services, Inc.**

| | | | |
|---|---|---|---|
| BORING STARTED | 5-10-05 | | |
| BORING COMPLETED | 5-10-05 | | |
| RIG | ATV | FOREMAN | DC |
| APPROVED | JCC | JOB # | 1-1571 |